Assessment of the effect of in-cycle degradation in behavior of mid-rise steel moment frames

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SUMMARY:
For many years, providing sufficient strength of buildings to avoid collapse prevention was the most important purpose in seismic codes. Nowadays, in-cycle strength degradation attracts the attention of many researchers as a real phenomenon. Recent investigations have shown that the effects of in-cycle strength degradation can be critical in determining the possibility of lateral dynamic instability in SDOF systems. In this research these effects in multi-degree-of-freedom (MDOF) structures have been evaluated by modeling 3, 5 and 7 story steel moment frame buildings with three types of connections (Pre-Northridge, Post-Northridge and Elastic-Perfectly-Plastic). By performing incremental dynamic analysis (IDA), effects of in-cycle strength degradation in instability of structures have been studied. Results of this study have shown that in-cycle strength degradation can highly increase the dynamic global instability of the steel moment frame structures.

Keywords: In-cycle strength degradation, Incremental dynamic analysis (IDA), Steel moment frame, Pre-Northridge connection, Post-Northridge connection.

1. INTRUDUCTIONS

Northridge earthquake (1994) was one of the most important events that produced basic variations in structural engineering applications and its philosophy because of widespread damages in beam-column connections, panel zone and none-structural members in buildings. Many analytical and experimental investigations were done by SAC-FEMA in accordance with pre-Northridge code requirements for Los Angeles and it was learned that most of pre-Northridge connections had brittle behavior. Lee and Foutch studied on steel moment frame performance with pre-Northridge connections. Yun et al. proposed reliability based performance level method for steel moment frames by using non-linear dynamic analysis. Therefore further research has been started that led to new types of steel moment connections with more strength and ductility than pre-Northridge connections. as a result of these researches, it has been seen that, strength and stiffness degradation effects in non-linear dynamic analysis can impress seismic response of structures. One the most important degrading behaviors were In-cycle strength degradation in which case, strength degradation occurs in the same cycle of loading. This phenomenon could outcome of P-Δ effects or Non-linear behavior or both of them. In some cases, In-cycle strength degradation, can led to lateral collapse of SDOF systems. In this research effect of in-cycle strength degradation in steel moment frames is studied.

2. IN-CYCLE DEGRATION

Cyclic behavior of members can affect stability of a structure. As mentioned before, one of the most important behaviors in steel moment frames is In-Cycle Strength degradation which strength degradation occurs in the same cycle of loading. There are two types of strength degradation, cyclic degradation and in-cycle degradation as depicted in figure 1. In accordance with Studies done by FEMAP440A on SDOF systems show that, in-cycle degradation can lead to lateral dynamic instability of the structures. In-cycle degradation can be due to P-Δ effects or nonlinear behavior of material or combination of both (ATC, 2005).

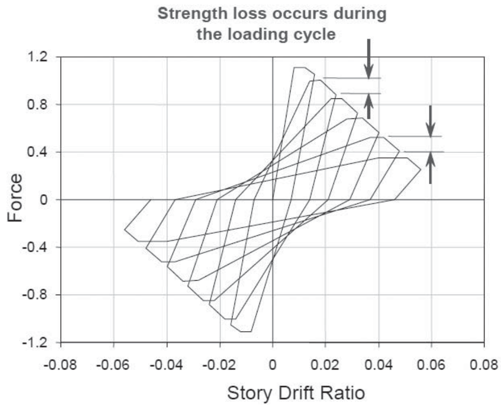

Figure 1. A typical in-cycle strength degradation behavior (ATC, 2005)

3. PRE-NORTHRIDGE CONNECTIONS

Two types of brittle connections,"a" and "b", are used. In order to introduce specifications of these connections, two kinds of connections are utilized. These connections characterized by force-displacement capacity boundaries that include strength degradation at 15% of the yield strength immediately after yielding, and their ultimate deformation capacity are 6% drift (Figure 2-3). These connections represent pre-Northridge welded beam-column connections in steel moment frame buildings which had large reduction in lateral resistance. Results of the experimental tests on pre-Northridge welded beam-column connections have exhibited kind of behavior that is similar to the one modeled in this section.

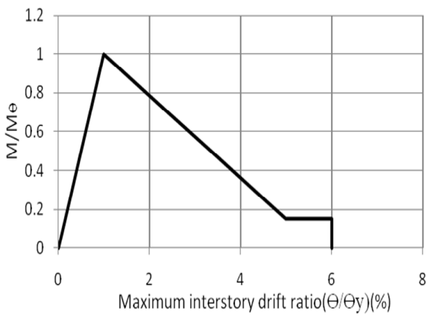

Figure2. Force-displacement capacity boundaries for connection 1b.

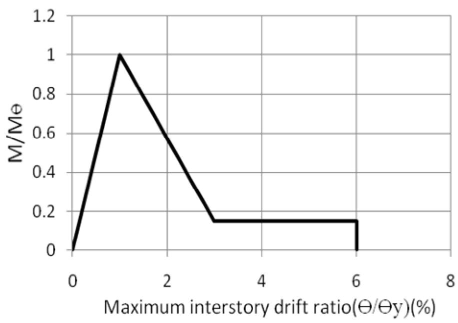

Figure3. Force-displacement capacity boundaries connection 1a.

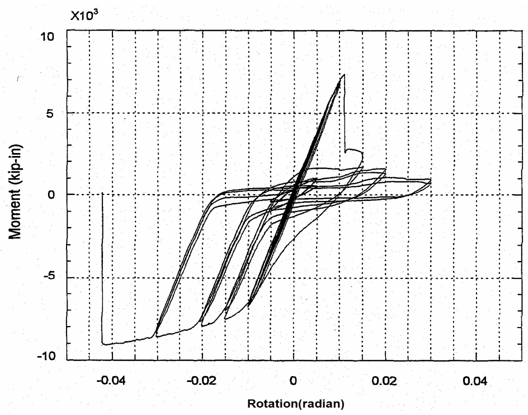

Figure 4. Hysteretic behavior from experimental tests on pre-Northridge welded steel beam-column Connections (femap440a)

4. POST-NORTHRIDGE CONNECTIONS

Like previous section, two connections for introducing the specification of ductile moment frame connections have been used. These connections characterized by force-displacement capacity boundaries that include strength hardening segment with a positive slope equal to 2% of the elastic stiffness and a strength degradation part that starts at 4% drift and ends at 6% drift. Also their ultimate deformation capacity is 8% drift (Figure 5-6). Residual strength plateau in a and b connections are equal to 45% and 80% respectively. These connections could represent special steel moment resisting frames with ductile (e.g., post-Northridge) beam-column connections and post-Northridge reduced-beam steel moment connections. Results of experimental tests on post-Northridge reduced-beam steel moment connections have exhibited a behavior that is similar to the one modeled here.

Figure 5. force-displacement capacity boundaries for connection 2b

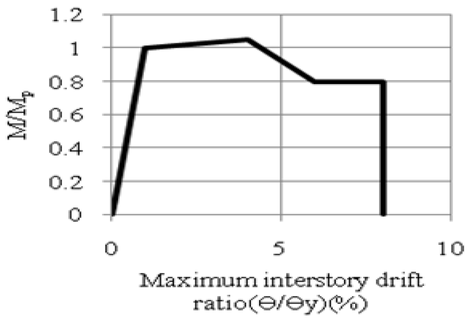

Figure 6. force-displacement capacity boundaries for connection 2a

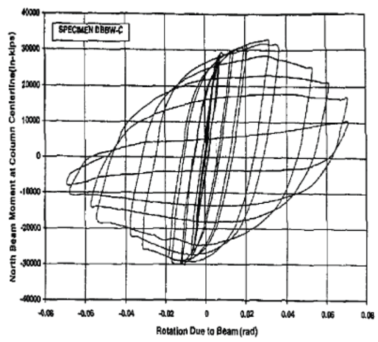

Figure 7. Hysteretic behavior from experimental tests on post-Northridge reduced-beam steel moment connections (FEMAP440A)

5. ELASTIC-PERFERCTLY-PLASTIC CONNECTIONS

In this kind of connection, there is no degradation and is used for comparing with other connections. These connections have Elastic-Perfectly-Plastic behavior. This kind of connection includes two parts, one elastic segment and other perfect plastic segment. The stiffness of the slope of perfect plastic segment is equal to zero. This connection is used just for comparing with other connections and it can be modeled for non-degrading structures. As shown in force-displacement capacity boundaries of these connections in Figures 8 and 9 the main difference in two connections are in ultimate deformation capacity. The ultimate deformation capacity in connection 3a is 7% drift and in connection 3b is 12% drift.

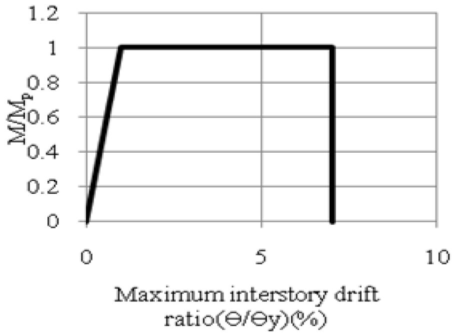

Figure 8. force-displacement capacity boundaries for connection 3a

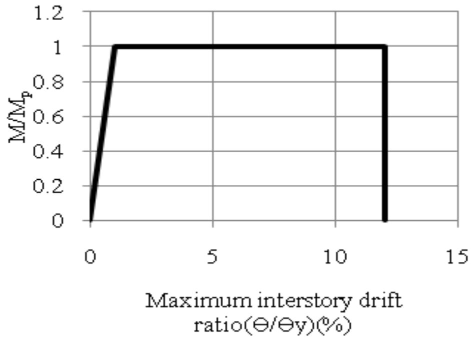

Figure 9. force-displacement capacity boundaries for connection 3b

6. STATEMENT OF PROBLEM

In this research a group of mid-rise steel moment structures include two-bay 3, 5 and 7-story building are utilized. The height of all stories and length of bays are 3.2m and 4m respectively. Buildings are located in a residential area in center of Tehran (Capital of Iran) with very high seismicity in accordance with Iranian practical building code (standard 2800-3rth edition) and the underlying soil is type III with average shear wave velocity of 175-375 m/s. And gravity loading is according to and seismic design has been done by Standard 2800. Also special criterions in 2800 standard for design of steel structures are considered. Dominant periods of two-bay structures are 0.81, 1.14 and 1.59 s.

Some assumptions here are:

- Frames in plane and height are regular therefore they are utilized as 2D frames
- Story Dead load is 1100kg/cm^2 and live load is 200 kg/cm^2
- Steel yielding stress is 2400 kg/cm^2

For analyzing, IDA analysis method is selected and then by scaling record motions in some levels, nonlinear dynamic analysis is conducted at each step and then IDA curve of maximum interstory drift ratio as structural response versus first mode spectral acceleration with 5% damping as intensity measure, are produced. For analyzing, OpenSees (Open System for Earthquake engineering Simulation) which is an open source finite element program is selected.

Nonlinear behavior is introduced by connections 1a, 1b, 2a, 2b, 3a and 3b. These connections are considered as concentrated hinges at beam-column connection zone. "zero-length element" in OpenSees is used to characterize nonlinear behavior of concentrated hinges. "Pinching4 material" is utilized to model moment-rotation relationship of connections type 1 and 2. "Steel01 material" is also used to model moment-rotation relationship of connection type 3.

P-Δ effect is considered to account for second order displacements of the structure during the analysis.

Panel zone inelastic deformations are neglected because it significantly affects on structural response.

in-cycle strength degradation effects are studied by comparison of IDA curves and assessing the effect of Post-Yield Behavior and Onset of Degradation in force-displacement capacity boundaries that are various because of difference in in-cycle strength degradation amount in types of connections.

7. ANALYSIS AND NUMERICAL RESULTS

For assessing in-cycle strength degradation, 6 kinds of connections, 1a, 1b, 2a, 2b, 3a, 3b, in 3,5 and 7 story buildings are considered. For conducting IDA analysis many none-linear dynamic analyses are conducted. At first, all of the fifteen ground motion records were scaled to multiple levels of intensity and then in each step nonlinear dynamic analysis is done, finally curves of structural response versus earthquake intensity are produced. Ground motion records are shown in table 1:

Table 1. Ground motion records

Raw	Year	earthquake	Magnitude	Station	Distance (km)	PGA (g)
1	1979	Imperial Valley	6.5	Delta	34	0.34

2	1987	Whittier Narrows	6	Hollywood Stor FF	25.2	0.221
3	1987	Whittier Narrows	6	Hollywood Stor FF	25.2	0.124
4	1989	Loma Prieta	6.9	57382 Gilroy Array #4	15.8	0.417
5	1989	Loma Prieta	6.9	57382 Gilroy Array #4	16.1	0.212
6	1989	Loma Prieta	6.9	47381 Gilroy Array #3	14.4	0.367
7	1989	Loma Prieta	6.9	1028 Hollister City Hall	28.2	0.215
8	1979	Imperial Valley	6.5	UNAMUCSD 6617 Cucapah	23.6	0.309
9	1999	Kocaeli, Turkey	7.4	Iznik	31.8	0.136
10	1999	Kocaeli, Turkey	7.4	Iznik	31.8	0.1
11	1989	Loma Prieta	6.9	1695 Sunnyvale - Colton Ave	28.8	0.208
12	1987	Superstitn Hills(B)	6.7	01335 El Centro Imp. Co. Cent	13.9	0.358
13	1987	Superstitn Hills(B)	6.7	01335 El Centro Imp. Co. Cent	13.9	0.258
14	1987	Superstitn Hills(B)	6.7	11369 Westmorland Fire Sta	13.3	0.172
15	1989	Loma Prieta	6.9	57066 Agnews State Hospital	28.2	0.172

IDA curves with various buildings and connections of type “a” and “b” of each connection by16%, 50% and 84% fractile curves are shown here:

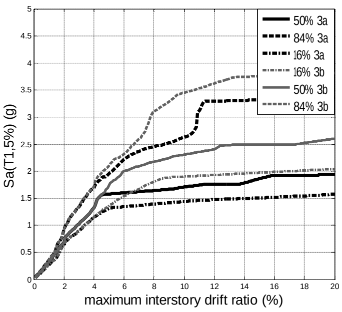

Figure10. IDA curves 3story with 3a and 3b connections

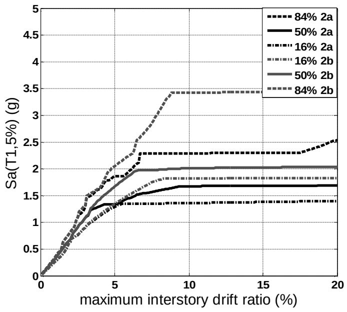

Figure11. IDA curves 3story with 2a and 2b connections

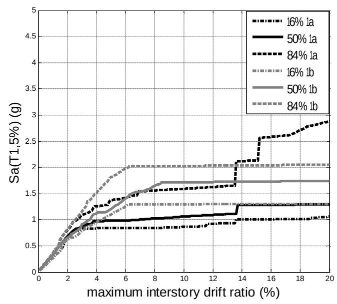

Figure12. IDA curves 3story with 1a and 1b connections

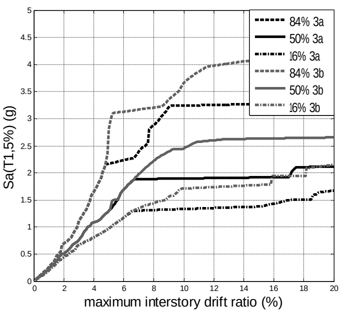

Figure13. IDA curves 5-story with 3a and 3b connections

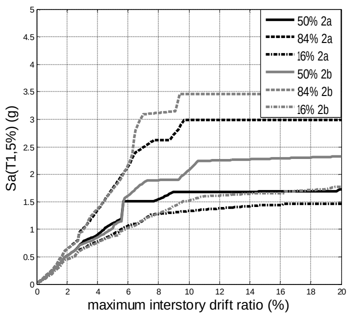

Figure14. IDA curves 5-story with 2a and 2b connections

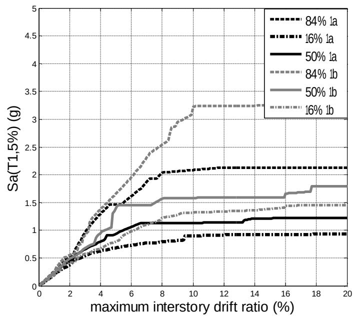

Figure15. IDA curves 5-story with 1a and 1b connections

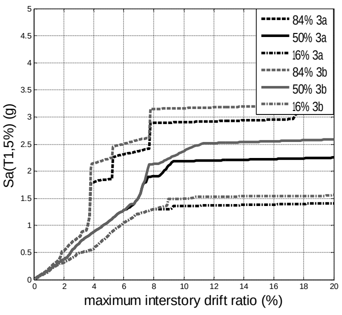

Figure16. IDA curves 7-story with 3a and 3b connections

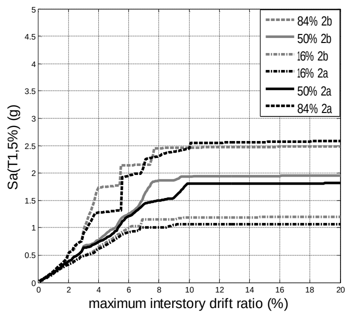

Figure17. IDA curves 7-story with 2a and 2b connections

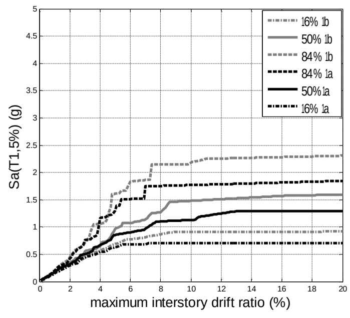

Figure18. IDA curves 7-story with 1a and 1b connections

IDA curves with type "a" and "b" connections are shown above; type "b" connection has better status from the aspect of in-cycle strength degradation. From the figures it's clear that, buildings with connections having In-cycle strength degradation have less ultimate capacity and lateral instability resistance and plateau section of curve is in lower drift response.

7.1. Comparison of IDA fractiles for studying the effects of Post-Yield Behavior and Onset of Degradation in force-displacement capacity boundaries:

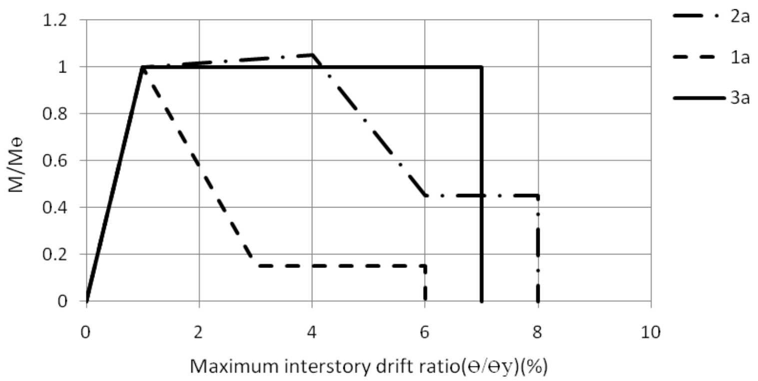

Figure19. Force displacement capacity boundary for 1a, 2a and 3a connections

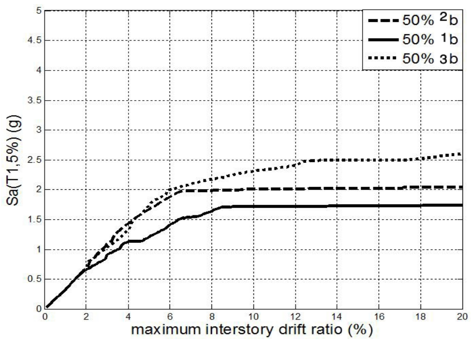

Figure20. comparison of connections 1b, 2b and 3b for three story building

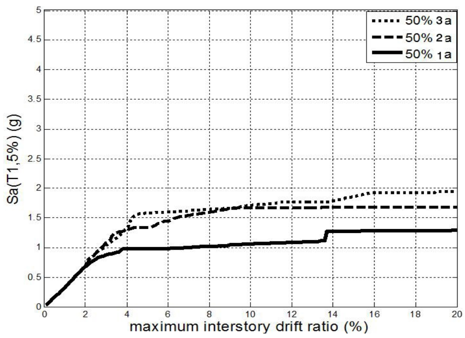

Figure21. comparison of connections 1a, 2a and 3a for three story building

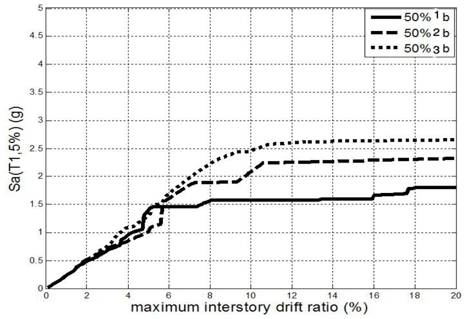

Figure22. comparison of connections 1b, 2b and 3b for five story building

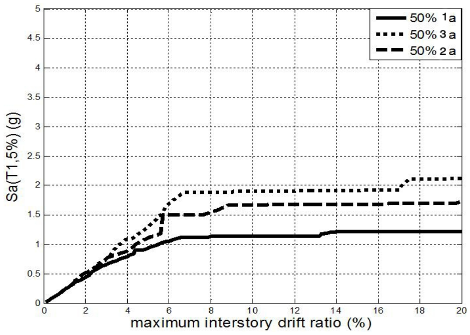

Figure23. comparison of connections 1a, 2a and 3a for five story building

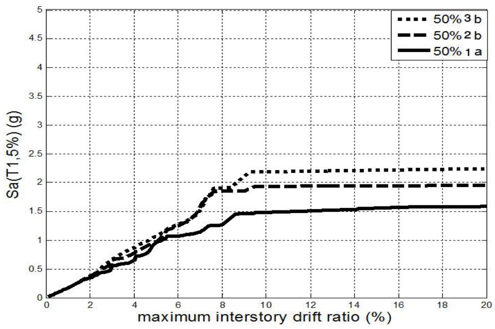

Figure 24. comparison of connections 1b, 2b and 3b for seven story building

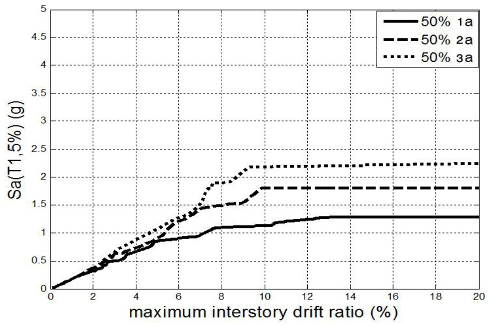

Figure 25. comparison of connections 1a, 2a and 3a for seven story building

8. CONCLUSION

- In this research, it has been seen that ultimate capacity of systems with high In-cycle degradation effects in connections (connections 1a and 1b), was less than the one with systems with low In-cycle degradation effects in connections (connection 2a and 2b) and also ultimate capacity of systems with non-degrading (connections 3a and 3b) connections. Systems with high In-Cycle behavior have the most ultimate capacity among all systems. Therefore it can be seen that in-cycle strength degradation can highly affect on ultimate capacity and instability resistance of system.

- In according to IDA curves it can be concluded that nonlinear dynamic response of a system is related to force displacement capacity characteristics of that system and Post-Yield Behavior and Onset of Degradation can highly affect on instability resistance of the system.

-According to previous conclusion that ultimate capacity of a system is related to force displacement capacity characteristics of that system, it can be concluded that with Retrofit Strategies, we can increase strength and ductility and instability resistance of the system.

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